Shaking the Foundations of Geo-engineering Education – McCabe, Pantazidou & Phillips (eds)

Equilibrium, strength, strain, dilation and superposition

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ABSTRACT: Some basic principles of mechanics are considered, using examples to show how they are important to practical design. In most of the cases considered, computer programs have been used to carry out calculations, with the user sometimes lacking a full appreciation of the problem or a proper understanding of the calculation and material model in use. Equilibrium is the fundamental requirement of many engineering calculations, and it is important to ensure that all forces can be transmitted into ground that is able to resist them; computer programs that consider only part of the equilibrium may be insufficient. Modern finite element programs are easy to use, but it is essential that users cling to a sound grasp of soil mechanics and behaviour, and constantly ask themselves if the model in use is suitable for the current task; in particular dilation is important in controlling the strength of undrained or confined soils. Engineering courses tend to concentrate on stress states and stress analysis; but strain is much easier to observe and sometimes gives important warnings of impending problems. Finally, the principle of superposition is carefully taught, with emphasis that it only applies in linear situations; but in practice this limitation is often forgotten.

1 INTRODUCTION

Computer-based calculations are now very widely used in geotechnical design. Unfortunately, the power of the computing is sometimes not matched by the understanding and knowledge of the users of programs. Furthermore, the limitations of computer software are sometimes accepted without proper consideration of the range of behaviours and events, the modes and mechanisms that are physically credible.

This paper is a plea that engineering students should be taught a sound appreciation of material behaviour and mechanisms, and encouraged to approach design with enquiring minds, not limited by the software that is readily available.

In Section 2, three situations are described in which the computations carried out for design did not consider the complete equilibrium of the construction. In Sections 3 and 4, analyses associated with two deep excavations are considered. These illustrate the significance of soil models, especially dilation, the importance of considering strains, and some possible pitfalls in assuming that superposition can be applied.

Some of the examples discussed below involve embedded retaining walls. Eurocode 7 contains a set of diagrams that show 30 different failure modes for retaining walls, of which 13 are relevant to embedded walls. Of these, the five shown in Figure 1 are considered within this paper.

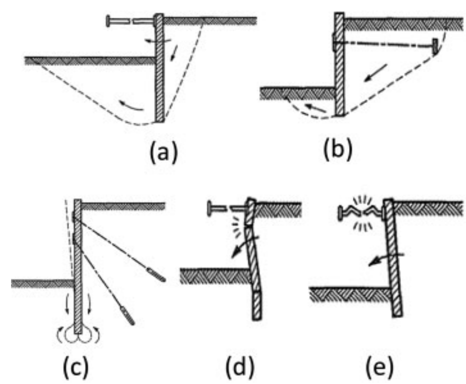

Figure 1. Five failure modes of embedded walls shown in Eurocode 7.

2 EQUILIBRIUM

2.1 *Introduction*

100 years or so ago, engineers had none of our present day computing aids and, in fact, few calculations were possible for geotechnical design. Instead, designers would draw, fairly slowly and laboriously, setting the proposed construction in its context, and in the course of doing so they would ponder whether what they were drawing would work successfully. In contrast, two problems can arise from modern computing, if it is not

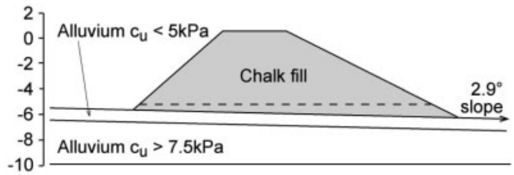

Figure 2. Embankment on very soft river muds.

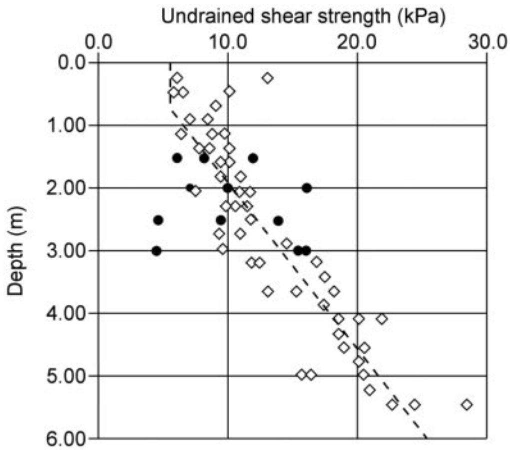

Figure 3. Measured shear strength of alluvium.

well used: (a) the particular item being designed is not seen in its wider context, so potential failure modes are missed, and (b) unreasonable proportions proposed for a new construction may not be recognized.

The following three examples illustrate these points.

2.2 *Embankment on alluvial foreshore*

Figure 2 shows an embankment proposed for construction on a river alluvial foreshore, which had a slope of about 2.9°. Figure 3 shows vane strengths measured in the mud, indicating a lower limit of about 5 kPa. The open diamonds represent data obtained higher up the foreshore where there would be more exposure to air. The solid circles show results from the actual location of the embankment; attention to the ground investigation report reveals that attempts to measure strength in the top 1.5 m had failed because no resistance was found.

The embankment was to be constructed of quarried chalk, with a basal layer of reinforcement. Computer analysis concentrated on the embankment and its reinforcement. Midway through construction, the embankment slipped sideways, remaining relatively intact, somewhat in the manner shown by the finite element analysis in Figure 4, which indicates a combined sliding and bearing failure. Analysis of this mode had not been considered, perhaps because it is quite complex and the significance of the extremely low strength of the muds was not understood, but mainly because of reliance on software that did not include this type of failure.

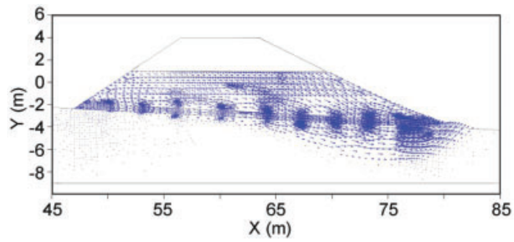

Figure 4. Probable failure mode of embankment.

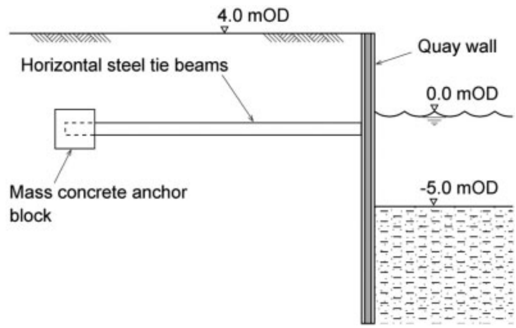

Figure 5. Old anchored quay wall.

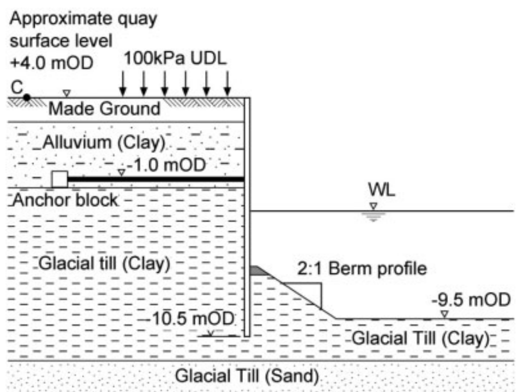

Figure 6. Anchored quay wall with proposed deepening of dock.

2.3 *Anchored quay wall*

Figure 5 shows the original cross section of an old anchored quay wall. To accommodate modern ships, it was required to have a greater depth of water in the dock, so it was proposed to dredge the seabed to give the section shown in Figure 6.

An engineer before the age of computing would probably have drawn Figure 6, noted the very small berm supporting the toe of the wall, and decided that the design looked unsafe. Modern engineers carried out computer analysis but were not alerted by the appearance of the cross section: perhaps they never drew it.

Many commercial computer programs are available for analysis of embedded retaining walls of this type.

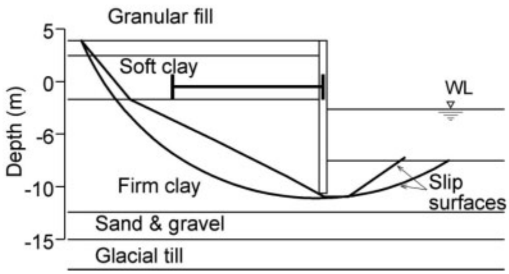

Figure 7. Slip surfaces for quay wall.

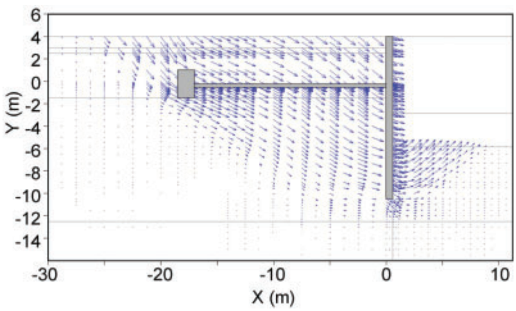

Figure 8. Failure mode shown by FE analysis.

Some consider only stability, whereas others try to compute also displacements and bending moments of the wall, and the tie force needed in the anchor. Factors of safety might be included in such calculations, but generally only the equilibrium of the wall and the earth pressures acting on it are considered. Stability calculations of this type were carried out and the revised design was considered to be acceptable.

However, when dredging was undertaken a crack formed in the quayside at the point marked C in Figure 6. Here again, the major problem was the use of analysis that concentrated on one mode of failure, in this case rotation of the wall about the tie (Fig. 1a), without considering other possible modes, such as shown in Figure 1b.

The strength parameters of the ground were not well known in this case. Figure 7 shows a circular and a non-circular slip analysis modelling the situation at which the crack formed. For the circular slip, a factor of safety of 1.19 is computed. The finite element (FE) analysis shown in Figure 8 became unstable at a factor of safety of 1.1, indicating a more critical failure surface than the circular slip. Figure 7 also shows a non-circular slip surface copied from the FE analysis, a factor of safety of 1.11, similar to the FE analysis. An advantage of the FE analysis is that, unlike the other analyses used here, it does not require a prejudgment of the failure mode and can accommodate the structural as well as the geotechnical elements.

2.4 *Vertical equilibrium*

Figure 9 shows an embedded wall intended for use at a riverside to allow construction of a storage area by filling behind the wall. The wall was to be supported by steeply inclined tubular piles, and was built in an old dock that had silted up over the last 100 years or so, giving very soft organic clay to around 10 m depth. The displacements of the wall were monitored. It collapsed during backfilling.

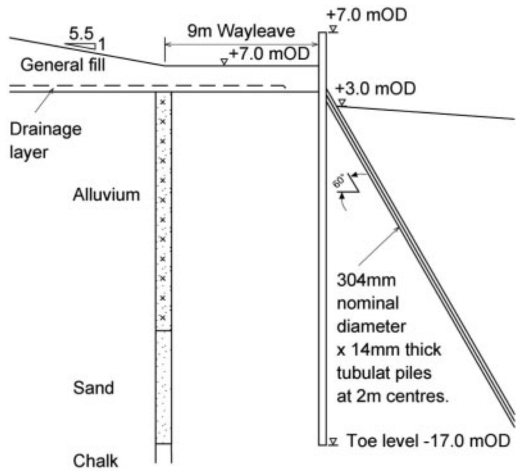

Figure 9. Sheet pile wall supported by tubular piles.

As noted in 2.3 above, many available computer programs can be used to compute the performance of this type of wall. Most of the simpler programs analyse the stability of horizontal earth pressures with wall bending moments and the horizontal components of the support forces; they do not consider vertical equilibrium, as illustrated by Figure 1c. As a result, a mode of failure resulting from inadequate vertical restraint was not properly considered, particularly noting the extremely soft nature of the very recent organic clay.

The monitoring showed that the wall failed by first moving upwards and outwards, normal to the inclined piles, until they were bent so much that they failed in bending. At that point, support to the wall was lost, probably due to local buckling of the piles, and the wall started to move downwards.

Once again, analysis had concentrated on use of particular software and had failed to take proper account of all possible failure modes.

2.5 *Teaching points*

Important points to note from the three examples in this section are:

- When using computer analysis, consider all possible failure modes, not only those modelled by the particular program available.
- For tied retaining walls, remember that the tie must be able to transfer the load to stable ground, and consider whether this will be achieved.
- For embedded walls, vertical equilibrium should not be forgotten.

- Note that very recently deposited clays may have extremely low strengths. Measured low values should not be assumed incorrect without very good evidence. Absence of measurements may indicate that the soils were too soft to give a reading.

3 NICOLL HIGHWAY STATION, SINGAPORE

3.1 *Introduction*

A major collapse occurred during the construction of the Nicoll Highway station, Singapore, in April 2004. During the following year, a public inquiry was held, which concluded that many aspects of the design and construction contributed to the cause of the collapse (Magnus et al 2005). The significance of some of these was disputed by involved parties, but it was generally agreed that the most important contributor was a steelwork error in design of the strutting system (Fig 1e). The inquiry concluded that some geotechnical issues also contributed to the collapse, of which three have been selected for their relevance to this paper. A more extensive discussion can be found in Simpson et al (2008).

The collapse occurred during the construction of a station box about 20 m wide and excavated to a depth of about 34 m. The cross section in Figure 10 shows that the excavation was largely in Singapore Marine Clay, which is soft to firm, normally consolidated or possibly slightly under-consolidated in relation to fill placed a few decades ago. The Marine Clay was underlain by layers of alluvial and estuarine clays of similar undrained strength, followed by much stronger Old Alluvium. The excavation was supported by concrete diaphragm walls, designed to extend into the Old Alluvium. Before excavation, jet grout struts had been formed at two levels, one above and one below the final excavation level. At the time of the collapse nine levels of strutting were in place, the upper jet grout strut had been removed, and excavation was underway to the tenth level. It can be seen in the plan of the area that collapsed (Fig 11) that the steel struts were generally placed in pairs. Strut pair S335 was instrumented with strain gauges and had inclinometers, I65 at its south end in the wall and I104 at its north end in a borehole just outside the wall.

The design was based on the results of finite element analysis using the program Plaxis. From this, anticipated wall displacements were derived, trigger values for monitoring were set, and wall bending moments and strut forces were derived.

3.2 *Irregular geology*

The design analysis was carried out for the cross section shown in Figure 10. It was assumed in analysis and specified on the drawings that the diaphragm walls were to extend 3 m into the Old Alluvium, to achieve toe support.

In reality, geology is never as simple and regular as could be inferred from Figure 10. In particular, the levels of the interface between the soft clays and the Old Alluvium varied sharply over short distances because the surface of the Old Alluvium had once been ground level and had been eroded irregularly by streams. For example, near inclinometer I104 there was a local dip in the surface and the design requirement for the walls to penetrate 3 m into it was overlooked, and the contractor recorded very little penetration. The geology on section A-A of Figure 11 was more like that shown in Figure 12.

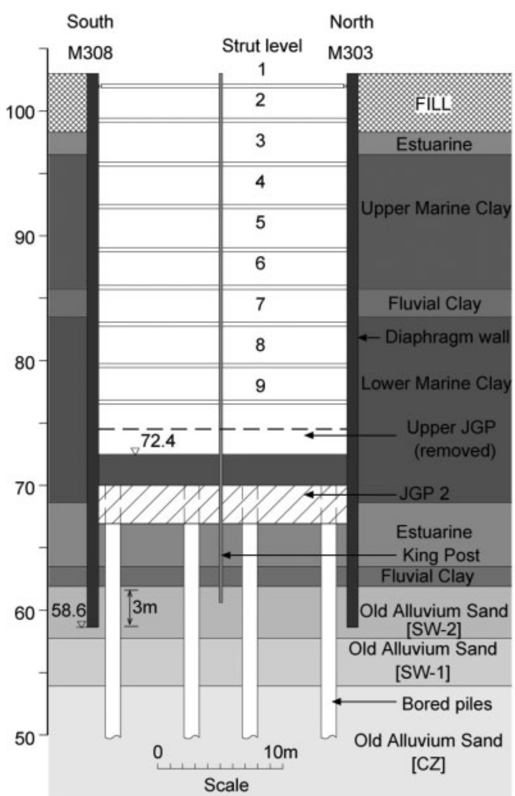

Figure 10. Nicoll Highway – design cross section.

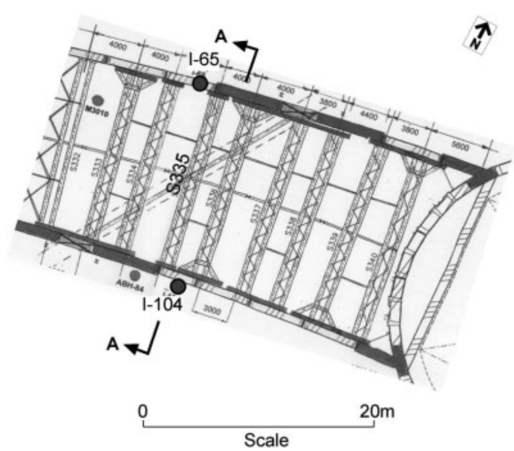

Figure 11. Nicoll Highway – plan.

As excavation proceeded, the measured displacements of the walls exceeded the trigger values at an early stage, particularly at I104. The designers re-calculated, progressively reducing the strength assumed for the soft clay in Plaxis, but without incorporating the information that the diaphragm wall had much less than 3 m penetration into the Old Alluvium. Consequently, their analyses failed to match the distorted shape of the wall actually observed, and computed bending moments were not realistic.

3.3 *Modelling undrained soft clay*

On the north side of section A-A, however, the wall had sufficient toe restraint in the Old Alluvium. Nevertheless, the observed displacements at Inclinometer I65 were roughly double those computed in the design, noted as “Method A” in Figure 13. During the inquiry, it was agreed that this was due to inappropriate modelling of the undrained behaviour of the soft clays.

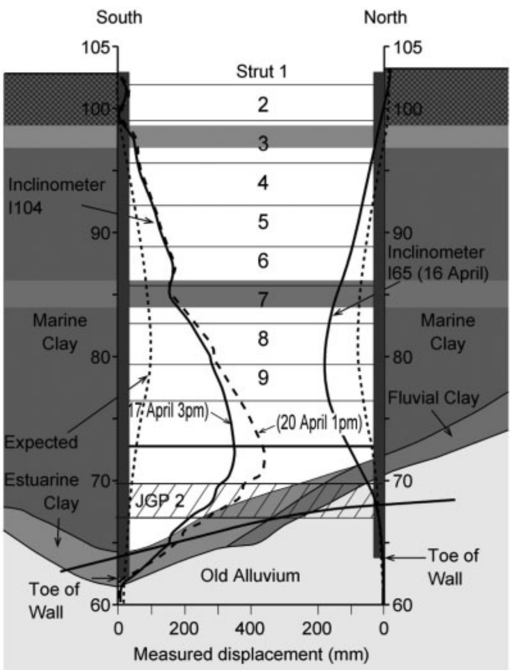

Figure 12. Nicoll Highway – displacements and geology.

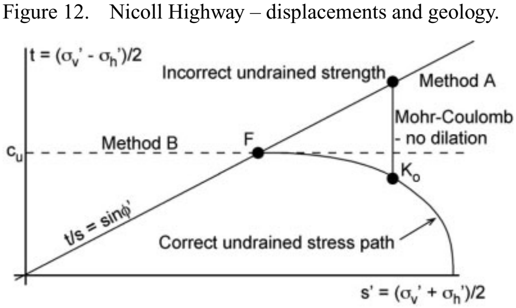

Figure 13. Modelling undrained behaviour – Methods A and B.

The soft clays were modelled using effective stress parameters, assuming linear elastic behaviour bounded by a Mohr-Coulomb envelope ($t/s = \sin\varphi'$). Figure 13 shows a shear stress/normal stress diagram for plane strain, in which the Mohr-Coulomb envelope is marked, together with a typical effective stress path for undrained behaviour of a normally consolidated clay. This path reaches failure at point F on the Mohr-Coulomb envelope with undrained strength c_u.

An element of soil in the ground would be expected to have in situ stress such as point K_0. From this point, the simplest elastic-Mohr-Coulomb model gives the vertical stress path K_0A for undrained behaviour. Because the strength is limited by the effective stress angle of shearing resistance φ', the undrained strength is overestimated by a factor of about 1.4.

This effective stress approach was known in the inquiry as Method A. An alternative approach for undrained behaviour, Method B, is simply to specify the undrained strength of the material, c_u, rather than the effective stress parameter φ'. For purely undrained behaviour this often a reliable and simple expedient, especially for normally consolidated clays. The undrained strength is computed correctly, but the computed pore pressures are still wrong. This becomes more problematic if consolidation is to be modelled following an undrained stage.

The result of the overestimate of undrained strength using Method A was that displacements of the walls were underestimated by a factor of about 2, as shown in Figure 14 for the north side (I65), which conformed more closely to the design stratification. This figure also shows that use of Method B with the anticipated undrained strengths of the clays gave a good

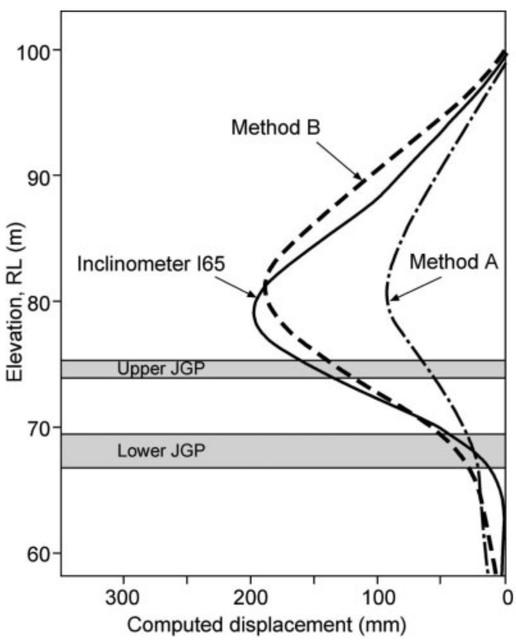

Figure 14. Maximum displacements for I65, computed and measured.

prediction of the displacements for I65. The computed ratio in bending moments for methods A and B was also about 2; this meant that the more correct predictions of Method B significantly exceeded the bending strength envelopes of the design, which was based on Method A (compare mode (d) in Figure 1).

From a teaching point of view, the main issue here is that engineers who will carry out geotechnical FE computations must understand enough about soil behaviour to judge the validity of available models for the problem in hand. In this case, an understanding of undrained stress paths was needed, essentially as they are controlled by dilation during shearing, negative for soft clays and tending to zero as the critical state is approached. The elastic-Mohr-Coulomb model used in the design had no dilation, and it is likely that the issue of dilation was not considered. In the author's opinion, a trained geotechnical engineer should realise that Method A gives an unreliable prediction of undrained strength and certainly is not suitable for normally consolidated clays.

It should be noted that since the time of Nicoll Highway Plaxis have altered the advice given in their manual about modelling undrained behaviour. However, their use of the term "Method A" is different from that used in the Nicoll Highway inquiry. Plaxis include within this term the use of more advanced non-linear models of soil behaviour, which may give better predictions of undrained stress paths. In the Nicoll Highway inquiry these were termed "Method D", and Method A was used only to refer to elastic-Mohr Coulomb models.

3.4 *Factors of safety*

The public inquiry concluded that the design of the retaining structures had incorporated inadequate factors of safety. Of particular relevance to geotechnical design, no safety or mobilisation factors had been applied to the strength of the ground and no check on the toe stability of the walls had been carried out. It would, in fact, have been possible to satisfy these two requirements simultaneously by carrying out "ultimate limit state" FE analysis with factored strength as required by Eurocode 7 with the UK National Annex. The inquiry also noted that the required values of factors of safety should be determined in relation to the severity of any possible collapse: in this case it caused very great danger to the general public using the adjacent 6-lane highway.

Engineering students should understand that there is uncertainty about the strengths of all materials, particularly those in the ground. Whilst not advocating the detailed teaching of codes of practice in university education, the author suggests that a clear understanding of factors of safety and their roles in design should be part of an engineer's academic education.

3.5 *Observations of strains*

Engineers are taught to think about stresses, forces and their equilibrium. In practice, engineers give too little attention to displacements and strains, despite the fact that these are much easier to observe and measure.

Tests on the type of grout used in the jet grout struts show that it fails in crushing at compressive strains typically around 1%, with a range of 0.5% to 1.5%. A simple calculation using the wall displacements such as shown in Figure 12 showed that, even several weeks before the collapse, the average strain over 20 m between the two walls was about 2% at the level of the jet grout struts. It could be expected that this strain would not be uniform and that some local strains would be much higher. In the author's opinion, this simple observation should have been a cause for major concern: either the jet grout was inadequate from the start, or it had been overloaded and crushed. In either case, its continued effectiveness was very doubtful. Despite this, in pre-collapse analyses to explain the large wall displacements, the designers continued to assume that the jet grout was performing as designed.

As the final collapse was developing, steelwork could be seen to be deforming alarmingly. Although engineers on site were conscious of this, they were more influenced by measurements from the strain gauges on strut pair S335, which indicated that the design loads were not being exceeded. A similar observation was made before the collapse of the Heathrow Express tunnels in 1994 (HSE, 2000): in this case the 9 m invert of the tunnel, supposedly of sound concrete, had been observed over a period of many weeks to have shortened by about 150 mm (about 1.7% on average).

Engineers need to be able to read the signs of unacceptable displacements and strains. An overconcentration in education on stress and strength, rather than the limiting strains of materials, might account for this failure.

3.6 *Superposition*

During the investigation, it was tempting to try to assign percentage effects to the various causes, and to assume that they could be added together. For example, x% for the Method A/B problem, y% for the jet grout problem, z% for the change of toe penetration in to Old Alluvium, so the combined effects would be $x + y + z\%$. However, it was shown that the magnitude of the effect of Method A/B changed significantly depending on the state of the jet grout and assumed stratification, and so on, in a highly non-linear way. In such circumstances, superposition of effects may be grossly misleading.

3.7 *Teaching points*

Important points to note from the Nicoll Highway example are:

- Engineers who will carry out geotechnical FE computations must understand enough about soil behaviour to judge the validity of available models for the problem in hand.
- A clear understanding of factors of safety and their roles in design should be part of an engineer's academic education.

- Besides understanding stress, equilibrium and strength, engineers should be taught to observe displacements and strains, and to consider whether these exceed the limits of the materials.
- In non-linear systems, superposition of effects may be grossly misleading.

4 DEEP EXCAVATION IN STIFF SOILS

4.1 *Introduction*

This example illustrates some important points related to use of finite element analysis in permeable, or semi-permeable, ground. A complex situation, involving initial design and a later dispute, will be simplified in order to bring out the points relevant to geotechnical education. The main point of contention was whether time-dependent coupled consolidation analyses were valid, or whether steady state seepage should be assumed.

The cross section in Figure 15 shows the temporary support structure for a proposed excavation about 30 m deep and 29 m wide. The retaining structure consisted of a soldier pile wall with sheetpiles driven as deep as possible between the soldier piles. If the sheetpiles did not penetrate to formation level, it was proposed to spray concrete on the exposed face between the soldier piles. Each level of excavation was expected to take 30 to 60 days.

The geological profile is shown in Figure 16, superimposed on part of a finite element mesh used for some of the analyses. The surrounding ground was mainly a rock formation that had been completely weathered to a residual soil comprising silty sand or sandy silt, sometimes with some gravel, generally somewhat variable in grading. SPT blowcounts showed that it was medium dense or stiff, trending towards dense or hard with depth.

Fill and sand of alluvial or marine origin overlay the residual soils to a depth of up to about 7 m. The residual soil was underlain by limestone, to which the soldier piles extended at this cross section.

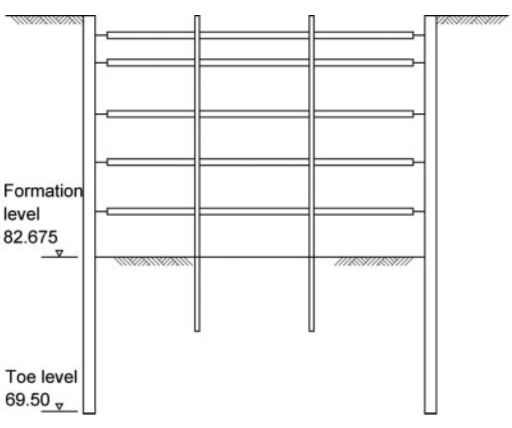

Figure 15. Deep excavation in completely weathered rock.

The properties adopted for finite element analysis are shown in Table 1, in which RS indicates residual soil and LST indicates limestone. Elastic-Mohr-Coulomb models were used for all the work described here. Further comments on the permeabilities shown in Table 1 are given below.

It is noted that this list does not include any parameters related to dilation of the soils or to the wall/ground interface friction. The author recognises that the use of c′ is a debatable topic, but it is not of importance in this problem.

4.2 *Permeability*

Figure 16 shows two profiles of permeability. Initially, a uniform permeability of 1E-7 m/s was adopted for the residual soil, as shown in profile A. However, there was concern that less permeable layers at depth might have an adverse effect on the behaviour of the excavation, so in later analysis the effect of reducing the permeability to 1E-8 m/s beneath excavation level was investigated, as in profile B.

The sand overlying the residual soil was thought to be more permeable and a permeability of 1E-6 m/s was adopted for this, with 1E-7 m/s for the fill.

The underlying limestone was thought to be potentially more permeable. However, it was intended that wells would be installed to relieve any free water at high pressure in the limestone, so the permeability allocated in analyses was as for the overlying residual soil.

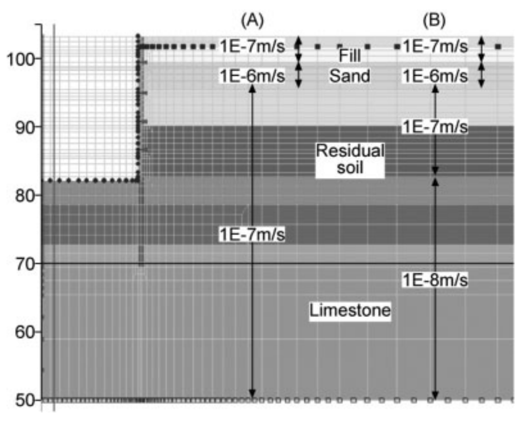

Figure 16. Permeability profiles A and B.

Table 1. Properties adopted for finite element analysis.

Soil	E′ kPa	ν′	c′ kPa	ϕ′ °	γ kN/m^3	k m/s
Fill	8,696	0.3	0	28	19	1E-7
Sand	8,696	0.3	0	30	20	1E-6
RS1	10,435	0.3	5	28	20	1E-7
RS2	38,261	0.3	10	28	20	1E-7
RS3	57,391	0.3	15	28	20	1E-7
RS4	107,826	0.3	15	30	20	1E-7
RS5	45,217	0.3	10	28	20	1E-7
LST	869,565	0.3	50	34	22	1E-7

Reflecting the proposed construction, the wall was generally modelled as impermeable or of low permeability above excavation level, but almost as permeable as the ground below that level. This was not readily achieved in Plaxis, however, because slip layers were used adjacent to the wall, so the full height of the wall was modelled as impermeable.

4.3 *A simple calculation*

In the absence of dilation, the time required for consolidation to be complete is a function of the permeability (k) and compressibility of the ground (m_v), and of the "path-length" (h), which is the distance water has to travel as the transient pressures dissipate. The compressibility and permeability terms can be combined with the weight density of water (γ_w) to give the coefficient of consolidation $c_v = k/m_v\gamma_w$. The time required for essentially complete consolidation is given by h^2/c_v.

From Table 1, a reasonable average Young's modulus for the ground is about $E = 50$ MPa, and the compressibility is roughly $m_v = 1/E$. The default permeability is generally 1E-7 m/s (profile A), and the weight density of water is taken as 10 kN/m^3. Since the water level was maintained near ground surface a reasonable average distance for the seepage path is about 20 m. For these values, the formula h^2/c_v is evaluated as about 9 days, at which point a steady state will apply. On this basis, the steady state will prevail within a period considerably less than the times required for any of the stages of the excavation, assumed to be 30 to 60 days. Any FE results that suggest a markedly different conclusion should therefore be regarded as suspect.

4.4 *Finite element analyses*

Finite element analyses were carried out for this problem by various parties using CRISP, Plaxis and, in investigations by the author's firm, Oasys SAFE. Results from Plaxis and SAFE have been in close agreement, provided strictly equivalent data are used.

FE analyses were initially carried out using both time-dependent coupled consolidation analysis (CCA) and steady state seepage (SSS) in CRISP. The bending moments computed for SSS were, critically, 55% greater than the CCA results. This result is contrary to the simple calculation reported in 4.3 above, which suggested that SSS would be established within the timescale of each stage of excavation.

To check CRISP, an SSS analysis was also carried out using Plaxis, in this case with an impermeable wall to full depth in both programs. As shown in Figure 17, the results from the two programs were in very close agreement. In the author's experience, such close agreement between two FE programs can usually be obtained, but only after extreme efforts to ensure that the input data to the programs are strictly equivalent, which usually takes several iterations. In this case, however, no such iterations had been undertaken.

Careful scrutiny of the data would have revealed two unintended differences: (a) In Plaxis it was assumed that the interface friction on the wall was half that of the ground (ie $\delta/\varphi' = 0.5$) on both sides of the wall, whereas in CRISP this was only applied on the retained side, with full friction on the excavated side. (b) Small differences in the geometry meant that the span of the section of wall at maximum bending moment differed by about 6%. Later analyses showed that these differences in data would have led to increases in bending moment from the SSS analyses (Plaxis assumptions relative to CRISP assumptions) of (a) 14% and (b) 12%. It is therefore clear, in retrospect, that the very close agreement for SSS analyses shown in Figure 17 between CRISP and Plaxis must be fortuitous and, in fact, could not be achieved with correct analyses of the data as used.

Figure 18 shows computed pore pressures for a point just below the level of the final excavated surface,

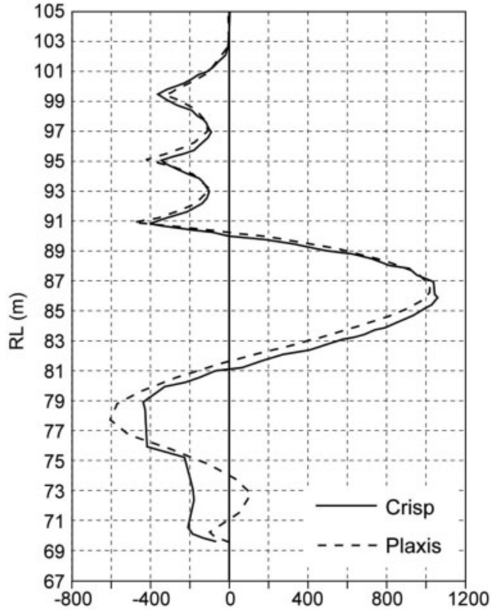

Figure 17. Bending moments computed initially for steady state seepage.

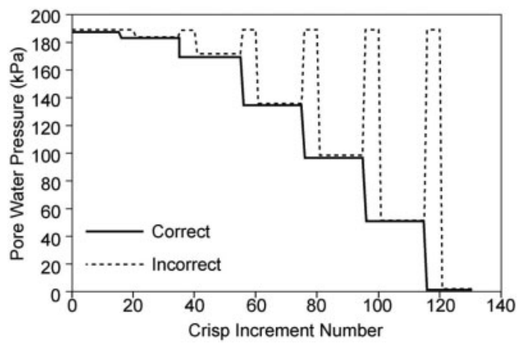

Figure 18. Computed water pressures just below final excavation surface.

in SSS analyses. The "correct" values reduce step-wise with each stage of excavation. However, investigation revealed that in the SSS CRISP analysis shown in Figure 17 the pore pressures were those marked "incorrect". By a data error, very high pore pressures were being specified at the excavated surface immediately after each stage of excavation. This should have led to instability, but the tangent stiffness method used in CRISP did not show this because only five "time increments", effectively iterations in time-independent steady state, were allowed at each stage. Had more iterations been allowed, larger displacements and bending moments would have occurred. The agreement shown in Figure 17 was entirely fortuitous, and very misleading.

Displacements and bending moments at critical stages are often the main results required from retaining wall analyses of this type. However, if the user is to understand the overall behaviour portrayed by the analysis, it is very important to inspect a wider range of output, at all stages of the computation. The author recommends in particular that engineers should examine displacement vectors (as in Figures 4 and 8) and principal stress fields to see the overall flow of displacement and stress, and also water pressures, usually best seen as contours.

4.5 *Dilation*

It was noted above that Table 1 does not mention dilation, a difficult topic that many geotechnical engineers prefer to ignore. For Mohr-Coulomb models, Plaxis and SAFE allow angles of dilation ψ to be specified by the user with default values of zero and a limit to the total amount of accumulated dilation. For its default value, CRISP assumes normality, treating the yield surface as a plastic potential, which, for a Mohr-Coulomb model, implies $\psi = \varphi'$. This is shown in Figure 19 by the vector of plastic strain increments $(\delta\varepsilon^{p}_{vol}, \delta\gamma^{p})$ for the plane strain case. All the CRISP analyses reported here use $\psi = \varphi'$, and it is not clear to the author whether this can be varied. Furthermore, dilation continues indefinitely in CRISP if shear strains become very large as shear failure occurs.

The actual dilational behaviour of the materials at this site was not known. It is thought that $\psi = 0$ is a reasonable, cautious value for design, and it is clear that $\psi = \varphi'$ is unreasonable, especially with indefinitely large shear strains.

It is not uncommon that expansion due to dilation is much greater than that due to elastic volumetric recovery. Thus the "h^2/c_v" formula is not applicable when there is significant plastic dilation.

Figure 20 shows a comparison between computed ground movements beneath the excavation for two SSS analyses with (a) $\psi = 0$ and (b) $\psi = \varphi'$, using permeability profile A from Figure 16. It can be seen that with dilation the heave of the surface is much larger. However, the displacement of the wall is similar in the two analyses. In fact, in all correct analyses using permeability profile A, for the cross section analysed

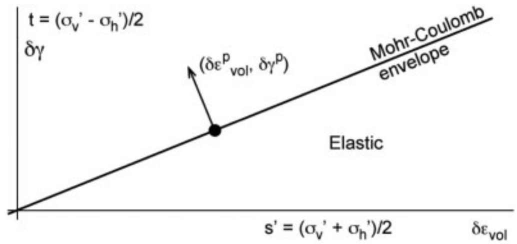

Figure 19. The Mohr-Coulomb envelope as a plastic potential.

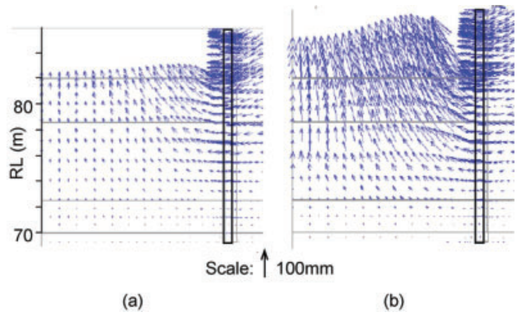

Figure 20. Ground displacements at final excavation for SSS (a) $\psi = 0$, (b) $\psi = \varphi'$.

dilation has very little effect on wall displacements and bending moments.

The situation is different for the lower permeability represented by profile B in Figure 16, however. Figure 21 shows computed water pressures for SSS, and two CCA analyses with $\psi = 0$ and $\psi = \varphi'$, both assuming 60 days for each stage of excavation. In this case, the water pressures for the SSS case and the CCA with $\psi = 0$ are similar, and the computed bending moments also agree very closely. However, the CCA with $\psi = \varphi'$ shows differing water pressures as they take longer to reach a steady state because the expanding ground absorbs more of the inflowing water. The computed bending moments for the SSS and CCA case with no dilation are 40% bigger than for profile A, while the bending moment for CCA with full dilation is much reduced, being less than for profile A.

In summary, the change from $\psi = 0$ to $\psi = \varphi'$ has little effect on bending moment for permeability profile A of Figure 16, but a big effect for CCA analyses with permeability profile B. Likewise, changing permeability profile from A to B causes an increase in bending moments for SSS or CCA with $\psi = 0$, but a decrease in bending moments for CCA with $\psi = \varphi'$. It is clear that the effects of these parameter changes are not additive: an assumption of superposition was shown to be highly misleading.

4.6 *A bug in CRISP*

During this work it was noted that contours of water pressure near the wall plotted by CRISP had the appearance shown in Figure 22. These imply very high

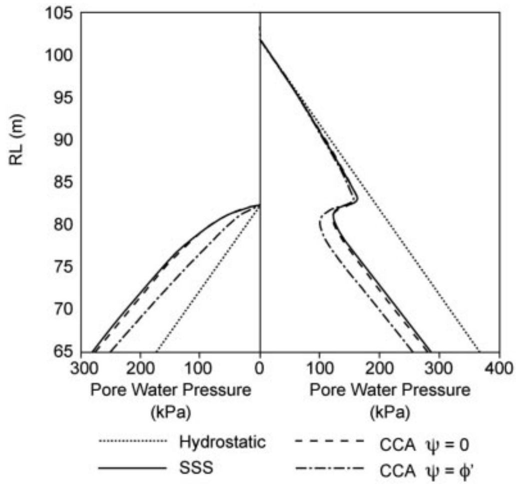

Figure 21. Computed water pressures for permeability profile B.

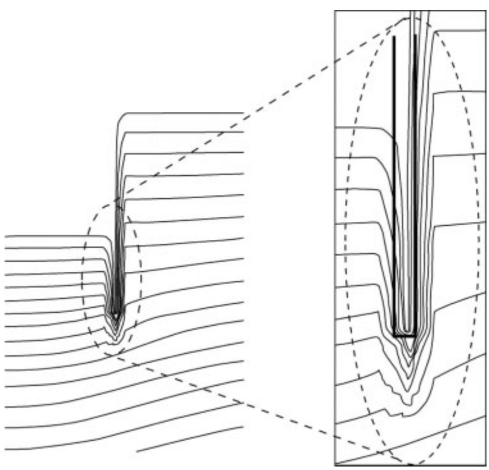

Figure 22. Impossible pore pressure contours from CRISP.

horizontal hydraulic gradients towards and away from the wall, both in the section above excavation of very low permeability and that below excavation of high permeability.

This situation is physically impossible and it transpired that it was caused by a bug in CRISP. It had the significant effect of reducing pore pressures in the layer adjacent to the wall and so increasing the available wall friction. The importance of inspecting computer output and questioning whether it is physically reasonable, is again emphasised, particularly in relation to water pressures.

4.7 *Teaching points*

Important points to note from this example are:

- When possible, simple calculations should be usued to check FE results. The h^2/c_v formula is frequently valuable.
- Users should inspect displacements, stresses and water pressures at all stages to understand the overall behaviour portrayed by an FE analysis, check for data errors and question whether the results are physically reasonable. Only then can the critical results, such as displacements and bending moments, be accepted.
- Dilation is a critically important parameter and must be understood by users of FE analysis.
- The effects of changing various parameters may not be additive in a simple way. Superposition should only be adopted when it is clear that the system is linear – often not the case in geotechnical engineering.

5 CONCLUDING REMARKS

On the basis of the examples considered, specific teaching points have been noted at the ends of the previous sections. The overarching points are:

- Engineers using computer analysis must not allow their thinking to be limited by available computer programs. They must ensure that all possible failure modes have been eliminated.
- Engineers must inspect computer results thoroughly, expecting to see them conforming to the basic principles of mechanics.
- Engineers who use FE analysis must have a sufficient grasp of soil behaviour, including dilation, to judge whether a particular model is adequate for their purposes.
- Engineers should be aware of the limiting strains of materials, as well as the strength limits.

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